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2. A summary of approximately 50 words must accompany the paper, a 300-word synopsis must precede it, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

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Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

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January, 1961

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(over)

Note.—Part 2 of this Journal is the 1961-4 Newsletter of the Surveying and Mapping Division.

Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

SURVEYS FOR THE LAKE PONTCHARTRAIN BRIDGE

By James M. Tuttle,[1] F. ASCE

SYNOPSIS

The overall survey control for the Lake Pontchartrain Bridge, having a length of some 24 mi over open water, presented challenging problems both before and during construction. The control measures used prior to construction consisted of triangulation in one form or another. Alignment and stationing during construction were attained through the use of towers, repetitive measurements, ingeniously designed templates, and other adjustable positioning devices.

INTRODUCTION

Lake Pontchartrain is a relatively shallow tidal body of water lying directly north of New Orleans. It is 43 mi long and 24 mi wide and varies in depth from 12 ft to 17 ft. On the south, the land adjacent to the lake is below sea level, and drainage is made possible by pumping. In this area lie heavily populated New Orleans and East Jefferson Parish. To the east and west are sea level marshes containing little development. On the north shore the land is higher and the marshes less extensive.

The need for a direct connection across the lake had been recognized and discussed for over a century. The recent growth of Greater New Orleans, Jefferson Parish, the growing importance of St. Tammany Parish, and the great increase in industrial development and truck traffic in southern Louisiana had

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made this connection imperative. In 1953 the voters of Jefferson and St. Tammany Parishes agreed to proceed with the design and construction of a bridge connecting the north and south shores of the lake. The general location of the bridge is shown in Fig. 1.

It was decided that the overwater length of this bridge should be a straight line, or "great circle" connecting P. I.'s on the north and south shores. The south shore P. I., which became known as the "South Shore Control Point," was to be established just west of Old Bayou Laurier, in anticipation of constructing the south approach road to join Clearview Avenue, then largely undeveloped. The north P. I. was to be located on the shore about one mile west of Mandeville. This location was selected because it avoided the swamps to the west and permitted excellent connections with the three branches of U. S. Highway 190 to the east, north, and west. This northern P. I. was termed the "North Shore Control Point."

BASIC HORIZONTAL CONTROL

Lake Pontchartrain is located in a region of humid atmosphere, where limits of visibility often become an important factor. Sight distances vary from a few feet in foggy weather to such rare days when the skyline of New Orleans might be discerned from high points in Mandeville. Because of the limitations on visibility and because of the problem of communications over this 24 mi stretch of water, it was decided to erect four triangulation towers in the lake.

At this time the U. S. Coast and Geodetic Survey had established lines of first order triangulation extending along the south, east, and west shores of the lake, but none along the north shore. The location of these lines is shown in Fig. 2. The need for additional control along the north shore was quite apparent.

The desire for accurate control along the bridge centerline and for strengthening the overall triangulation network was mutual, with the engineers and the U. S. Coast and Geodetic Survey, who accordingly entered into an agreement whereby that agency would extend their net across the lake and complete the system along the north shore, the cost of which was borne by the engineers. The engineers designed and supervised construction of the lake towers and selected their location as well as the location of the triangulation points on each shore adjacent to the bridge.

The first job was to locate the position of the lake towers. On a chart was drawn the centerline of the proposed bridge. The towers were plotted at about 8 mi from each shoreline, and 3 mi from each side of the tentative centerline of the bridge. Angles to these towers from aero beacons, lighthouses, and radio towers were determined graphically. By the use of a sextant, a motor launch was positioned at the location of each predetermined tower location and the spot marked by a 2 in. x 2 in. pine post, to which was affixed a bamboo pole festooned with yellow flagging.

Immediately following this work the construction of the towers was begun. Each tower consisted of a three pile instrument platform surrounded by a four pile supported working platform. The working platform consisted of one level 15 ft above the water surface to serve the instrumentman, and another level about 23 ft above the water surface on top of which were mounted the directional and navigation lights. The details of this tower are shown in Figs. 3 and 4.

Following completion of the towers the surveying crews began their work of triangulation. Steel Bilby towers were erected at chosen locations along the

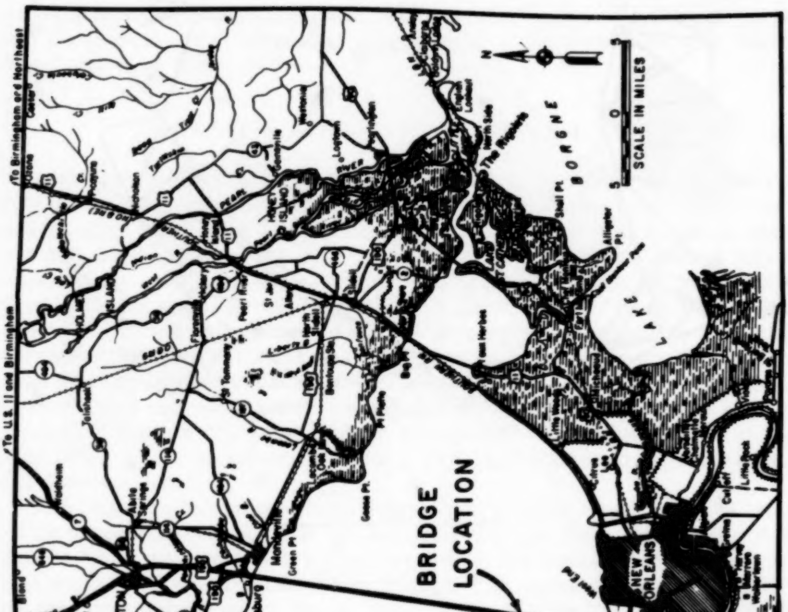

FIG. 1.—BASIC HORIZONTAL CONTROL

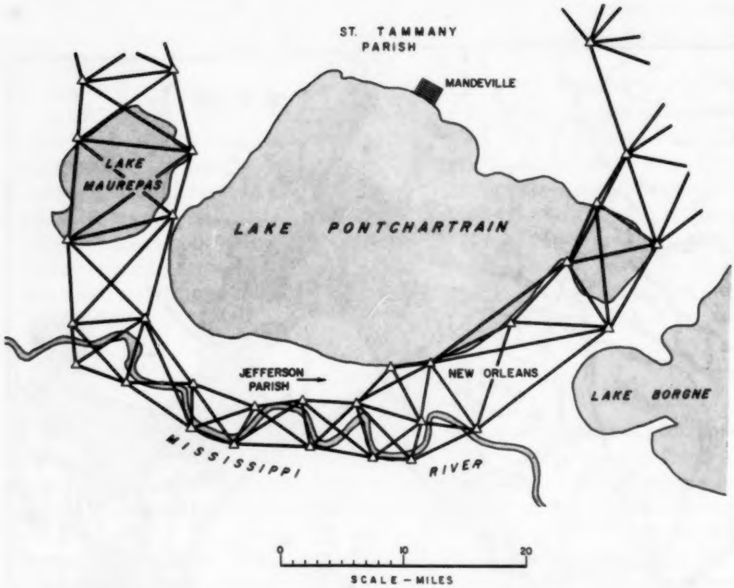

FIG. 2

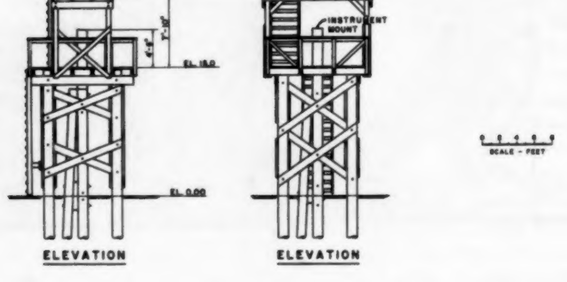

FIG. 3.—LAKE TRIANGULATION TOWER

northern perimeter of the lake, and from existing stations to the east and west a new net was extended connecting them. From these new control points the net was then carried southward across the lake, using the lake towers to tie in with the existing triangulation stations in New Orleans and Jefferson Parish on the south shore. The network of triangulation around the lake after this work was finished is shown in Fig. 5.

LOCATION OF THE BRIDGE CONTROL POINTS

It was now time to place permanent monuments at each of the two shore control points. On the north shore a 2 in. steel pipe was driven deep into the ground and plugged with wood into which was driven a survey tack. No tower was erected at this point. On the south shore the location of the control point was under study. After due consideration, the original location of the South Shore Control Point was abandoned, and a new point, called "South Shore Control Point No. 2," was established. This new point was located on the centerline of Causeway Boulevard (then named Harlem Avenue), which had a 180 ft dedicated right-of-way from the lake to the Mississippi River, and could more feasibly be developed into an arterial highway. A brass bolt embedded in a concrete monument marked this point. As the station was located just back of the lakeside levee, it was necessary to erect a 35 ft high wooden tower for unobstructed observations.

The final position of the North Shore Control Point was located by triangulating a quadrilateral formed by the control point, Lake Station No. 3, Lake Station No. 4, and Lewis. In a similar manner, the location of South Shore Control Point No. 2 was determined by triangulation, using a quadrilateral formed by the control point, Lake Station No. 1, Lake Station No. 2, and WWL Radio Tower. These triangulation points are shown on Fig. 6.

SOUNDINGS

On the lake a crew was engaged in making soundings along the bridge centerline. The soundings were made in the following manner:

On a boat were instrumentmen with sextants who sighted on visible triangulation towers. On each side of the boat sonic-type depth measuring devices were mounted. In the cabin was a plotting board. The speed of the boat was regulated so that observations taken at 2 min intervals would result in a spacing of soundings of about 1000 ft to 1200 ft. Upon a signal from the recorder, the two instrumentmen would call out their angles. Simultaneously the plotting board operator would push the "fix" buttons on the depth recording instruments, record the time on the moving charts, plot the position and, if necessary, give the pilot a correction for this course.

LOCATIONS FOR BORINGS

A series of deep borings were made at one-mile intervals along the centerline of the proposed bridge. At the time this work was to begin, the positions of the shore control points had been computed and stationing established for


KEEP OFF

FIG. 4

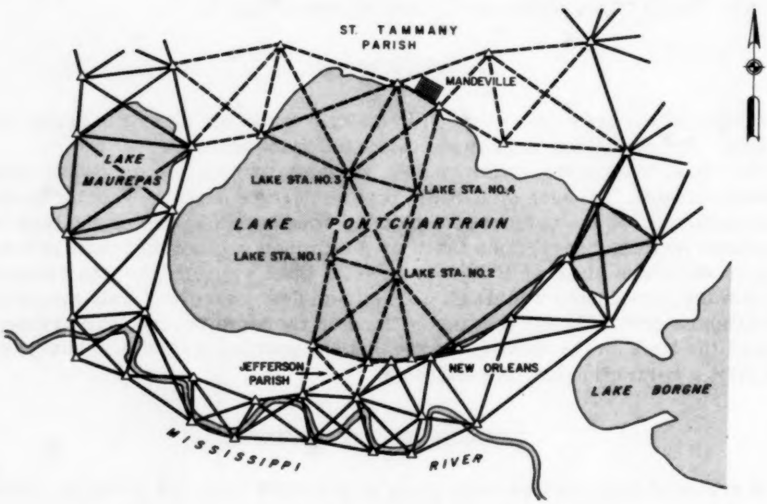
ST. TAMMANY
PARISH
MANDEVILLE
LAKE
MAUREPAS
LAKE STA. NO.3
LAKE STA. NO.4
LAKE PONTCHARTRAIN
LAKE STA. NO.1
LAKE STA. NO.2
JEFFERSON
PARISH
NEW ORLEANS
LAKE BORGNE
MISSISSIPPI
RIVER
0
10
20
SCALE - MILES

FIG. 5

the entire project. The position of the station on the bridge centerline at each of these boring locations was computed, and using these computed positions, angles to them were determined from selected triangulation stations. A map showing these various angles was issued to each party to be engaged in the field. This map (with the numerical value of each angle not shown) is shown in Fig. 6. Optional, or alternative, resection angles were given to some boring locations so that restrictions on limits of visibility might not slow down the work.

The field work was carried out by the engineers using two instrument crews, one at each triangulation station. The procedure was for each instrumentman to turn the predetermined angle, and "jockey" a helicopter over the location of the boring. The helicopter would then drop a weight to which was fastened a line and colored buoy. By the time three points had been located in this fash-

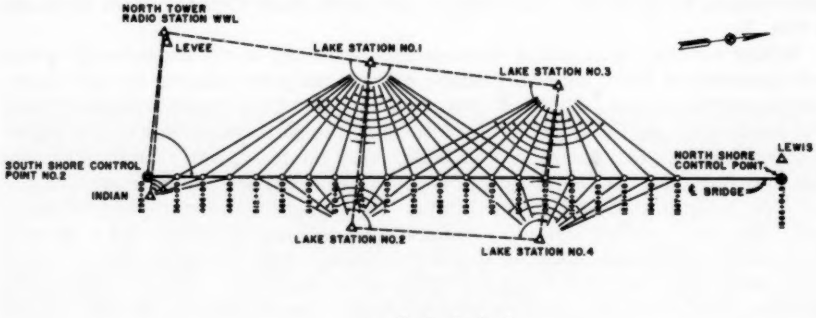

FIG. 6.—BORING LOCATIONS

ion, the ceiling lowered and this method was suspended. Had the weather cooperated, the entire work might have been completed in one day. As time did not permit waiting for favorable weather conditions, a diesel launch was used for the rest of the work.

Communications were carried out using two-way UHF radios. Power was furnished by gasoline-driven generators at the triangulation stations and by batteries on the launch and helicopter.

As an indication of the importance that weather played in this work, it is interesting to note that approximately 80 hr were required to complete this

operation, and that 70% of this time was non-productive because of poor visibility, rough water, or a combination of the two.

CONTROL DURING CONSTRUCTION OF BRIDGE

The bridge to be built consisted of a series of pretensioned, precast concrete slabs resting on precast pile caps 56 ft center to center. The pile caps were supported and rigidly concreted to two 54-in. cylindrical, hollow, prestressed concrete piles. From abutment to abutment, the length was approximately 23.8 mi all over water. At about 4 mi from each end and in the center, raised profiles or "humps" were to be built to provide for the uninterrupted passage of small craft. At approximately 8 mi from each end, bascule lift spans were to be provided, and at about 9 mi from the south end, a turnaround was planned. With minor exceptions the entire structure was designed for an assembly-line type of construction.

The basic control problem during construction was that of keeping the center of the bridge on the line between the shore control points and assuring that the precast units fitted properly. To assure accuracy of alignment a 90-ft Bilby tower was erected over each control point. Following this, five temporary three-pile platforms were erected in the lake along the route. By sighting from the Bilby towers, the exact centerline was marked on each of these platforms. The points marked on the platforms were subsequently used for control as the construction progressed. The tower at the North Shore Control Point is shown in Fig. 7.

Bridge erection proceeded continuously from the north shore to the south and consisted of three general phases; driving the piles, placing the pile caps, and placing the spans. The first two of these operations required careful control measures. As the permissible tolerance in pile location was on the order of about 3 in., a giant template was built. This template, some 26 ft in length, was lowered over the two sets of piles previously driven, and by means of adjustable, hydraulically operated clamps, it was aligned and clamped into position. The final position of the template was checked by means of a transit mounted on a platform fastened to the previously installed piles. The two new piles were then inserted in guide openings at the south end of the template, plumbed, and driven to the required bearing. The template was then raised and moved forward to its next position and the procedure repeated. This operation is shown in Fig. 8.

The cap placing operations followed the pile driving by one-quarter to one-half mile. The vertical and horizontal positioning of the caps was one requiring an accuracy of a small fraction of an inch. Here again the alignment was set by a transit mounted on the caps already in place and the distance measured by means of a calibrated chain from several pile caps back. This operation is shown in Figs. 9 and 10. Until the grouting operation, which permanently connected the precast cap to the piles, was completed and the concrete had set, the cap was held in place by an adjustable framework clamped to the piles. Vertical control was maintained by precision levels emanating from bench marks on the north shore. The placement of the deck slabs was more of a construction technique than a survey control problem, as the anchor bolts had been yard cast to the caps. The precision used in erecting the pile caps and the fine tolerances maintained at the casting yard paid off many-fold when the 200-ton slab

FIG. 7

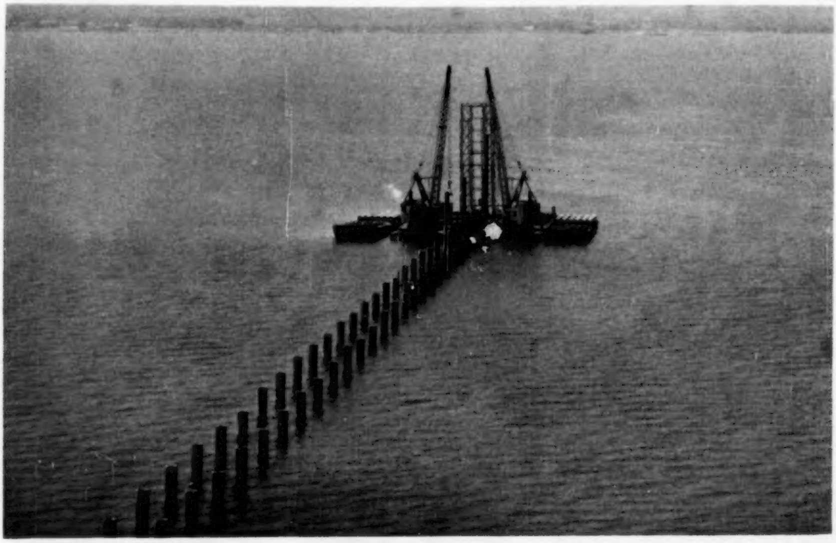

FIG. 8

FIG. 9

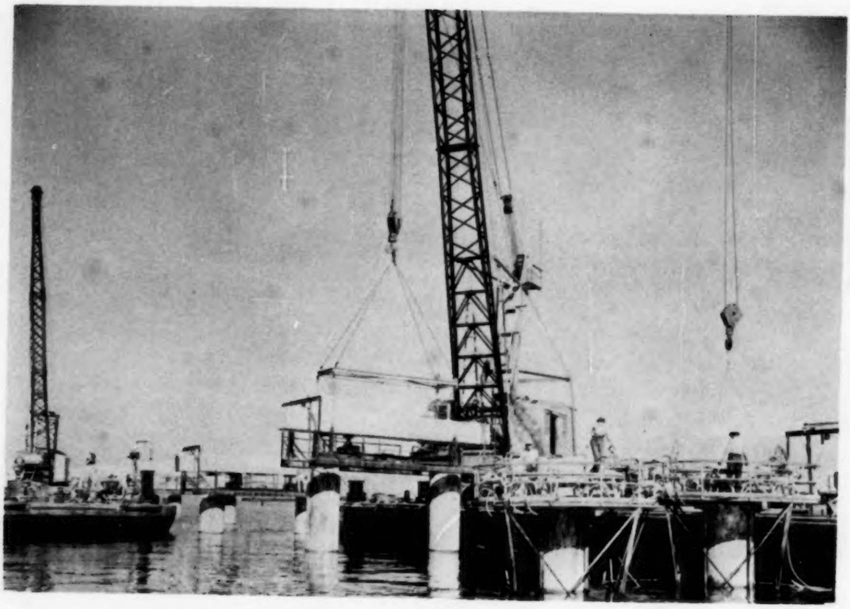

FIG. 10

units were lowered to a snug fit on their bearings, and produced a smooth, even joint on the riding surface. A view of the completed bridge is shown in Fig. 11.

The surveys prior to construction had indicated the presence of a 14 in. high pressure subaqueous gas pipeline crossing the project centerline at an angle of about 20°, about six mi from the north shore. As it was feared that the standard 56 ft bent spacing might result in piles being driven dangerously close to the location of this pipe, it was decided that a special span be designed when construction reached this point. By great fortune, it was found when the work

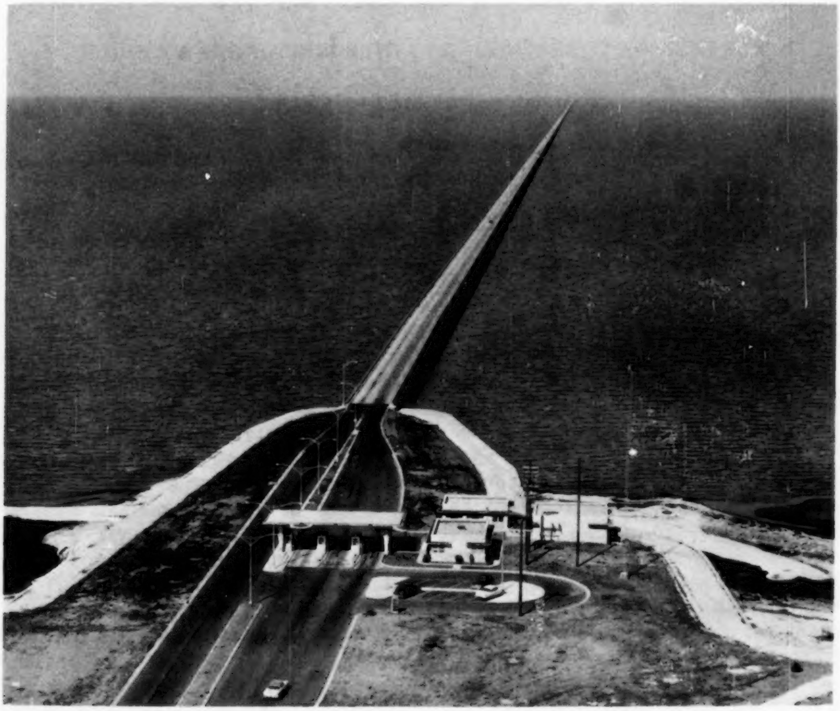

FIG. 11

reached this location the pipeline passed almost exactly between the two closest bents, and the special design was not required.

CONCLUSIONS

Despite the adversities of weather, and sometimes rough water, the survey measures employed in controlling the alignment and stationing for the Lake Pontchartrain Bridge were quite effective. The bridge was built continuously,

at a rate of approximately one-half mile per week from the north shore to the south with stationing carried forward with the construction. The location of the south abutment was planned at Station 306+00 and its actual location came out at Station 305+90.5, a difference of 9.5 ft. This small error of closure, which amounted to less than 1/16 in. per span for the 2,246 spans, resulted in an overall accuracy of about one in 13,500, or within the limitations of second order accuracy, was a tribute to the precision construction methods and survey control used in erecting the bridge.

Journal of the
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HIGHWAY AND BRIDGE SURVEYS: CONSTRUCTION SURVEYS

PART I—TRIANGULATION

Progress Report of the Committee on Engineering Surveying
of the Surveying and Mapping Division

GENERAL

The preliminary survey control system for a bridge project will provide the necessary information for design of the structure which need not be extremely precise. On the other hand, when the construction stage is reached, particularly for long water crossings, a highly precise horizontal control system is necessary, a system that will accurately and quickly enable location of various widely scattered piers and other component parts of the bridge.

For the long water crossing, this control generally will entail a well designed system similar to that which the United States Coast and Geodetic Survey (USC and GS) uses for its geodetic triangulation, except that better than first-order precisions will be required.

The precise location problem for the bridge construction is more a matter of accurately locating component parts of the bridge with respect to one another. In other words, exact location and span distance between two main piers of a cantilever bridge is primarily important although the location of these two piers and the remainder of the bridge within a certain county of the state is of secondary importance.

DESIGN OF TRIANGULATION SYSTEM

It has been pointed out[1] that existing survey control should be used as much as possible and that the preliminary control system should be designed with the possibility of incorporating it into the final location control system. This incorporation may not be possible for all projects, but in the majority

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1 "Highway and Bridge Surveys: Preliminary Bridge Surveys," Committee on Engrg. Surveying, Vol. 84, SU 3, Proceedings Paper No. 1842, ASCE, November, 1958.

of cases it is generally feasible to provide for certain control points to be common to both the preliminary and construction control systems. This coincidence results in a direct tie between the preliminary and final survey work and can also result in savings in survey time.

In general, the basic horizontal control system for a bridge crossing consists of a network of triangulation made up of one or more quadrilaterals, with the vertices or stations determined by physical survey control points or "paper" reference points, some of which are on the bridge alignment as points of intersection (P.I.'s), points of curvature (P.C.'s), or points of tangency (P.O.T.'s). A known distance must be introduced into such a system to permit the computation of the sides and adjustment of the angles of the network. The simplest approach to this is as illustrated by Fig. 1, in which two measured base lines are established on opposite shores of the crossing and

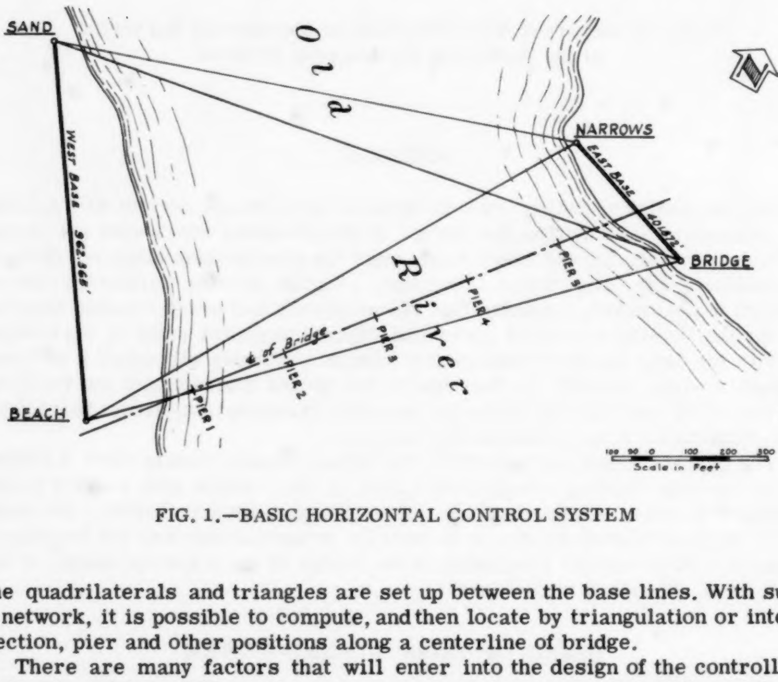

FIG. 1.—BASIC HORIZONTAL CONTROL SYSTEM

the quadrilaterals and triangles are set up between the base lines. With such a network, it is possible to compute, and then locate by triangulation or intersection, pier and other positions along a centerline of bridge.

There are many factors that will enter into the design of the controlling network and invariably make necessary a system somewhat different from that of Fig. 1. For better than first-order accuracy, a triangulation network must have adequate strength of figure. More complete examinations[2,3] of strength of figure have been presented.

In general, the network should be designed to consist of acute triangles whenever possible, rather than obtuse triangles. The intersection of lines of sight or directions that will be used later for main-pier locations should give strong fixes intersecting at angles in the order of 90° and not at relatively

[2] "Manual of Geodetic Triangulation," USC and GS Special Publication, No. 247.
[3] "Technical Procedure for City Surveys," Manual No. 10, ASCE.

small or large angles. The control points of the network, particularly in the water where islands or specially-constructed towers will be needed, should be located to result in optimum lengths of the lines of sight that will be used in locating the piers. Generally, with modern theodolites, sights of 3,000 ft to 5,000 ft in length are desirable. Lines of sight longer than these in areas with large water expanses will become troublesome during periods of excessive radiation and refraction. For the routine construction location problem, it is not desirable to have sights of such length and direction as to require observation at night with lighted targets. Night work for much longer lines of sight is quite common and often desirable during the measurement of the angles of the entire network.

The type of bridge will obviously influence, in large measure, the shape of the network for horizontal control. The suspension type of bridge with two main piers located near the opposite shore lines will generally require an accurate crossing distance, but pier location will be relatively simple and confined to the two main piers and possibly smaller approach piers near or on the shores. The typical arch or cantilever type of bridge often has long approaches made up of small-span trestle construction leading to longer-span steel girder and truss construction. This type of bridge, particularly when the alignment is curved, presents the problem of locating many piers with construction work starting at several different places simultaneously.

The $8\frac{1}{2}$-mile San Francisco-Oakland Bay Bridge in California extends more than 4 miles over the surface of San Francisco Bay. The bridge crosses Yerba Buena Island near the center of the bay, thereby dividing the main structure into two major parts.

The West Bay Structure consists of twin suspension bridges with a common central anchorage, and the East Bay crossing includes a high-level cantilever, a series of 509 ft and 288 ft truss spans and a hydraulic fill. The portion across Yerba Buena Island consists of a tunnel with reinforced concrete and steel viaduct approaches.

The construction of this crossing was controlled by an elaborate triangulation net with a base line on both the San Francisco and Oakland shores, and with two check base lines on Yerba Buena Island as illustrated by Fig. 2.

The Richmond-San Rafael Bridge spans 4 miles across the northern end of San Francisco Bay and is almost entirely over water. Approximately three-fourths of this crossing consists of a series of truss spans with identical high-level cantilever spans interposed across the two main channels. The remaining portion is composed of girder spans and a prestressed concrete trestle.

In order to obtain desirable locations for the base lines and to avoid intervening hills, thirteen stations were included in the original triangulation net. After the final adjustment, six of these stations were abandoned and the remaining seven were used for construction control.

The control net designed for the proposed Southern Crossing of San Francisco Bay furnishes an excellent example of the use of control towers (shown on Fig. 4 as triangulation platforms) located in the water. The proposed 8 mile crossing consists of a subaqueous tube under the channel flanked by nearly 5 miles of pile trestle and a hydraulic fill.

The primary triangulation net for this crossing, as illustrated in Fig. 3, is composed of seven stations, and the net adjustment is based on an inversed computation of the distance between the USC and GS first-order stations, Central and Farm 2, often referred to as "air base" or computed base.

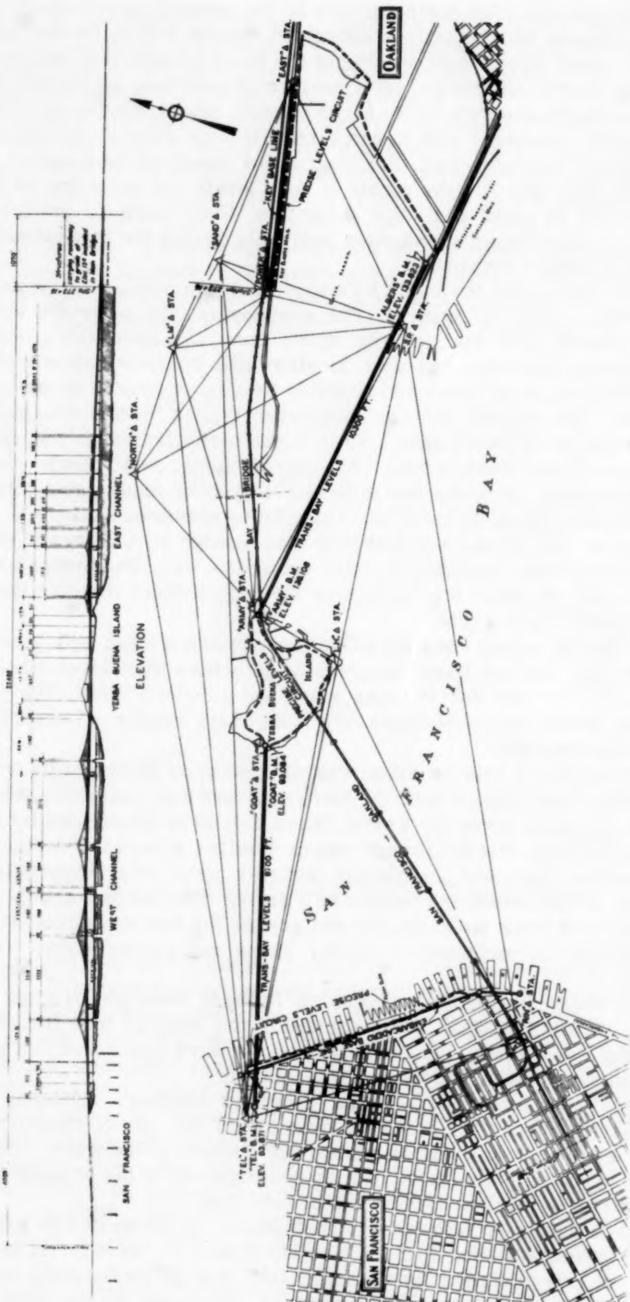

FIG. 2.—CONTROLS FOR CONSTRUCTION SURVEY OF SAN FRANCISCO-OAKLAND BAY BRIDGE

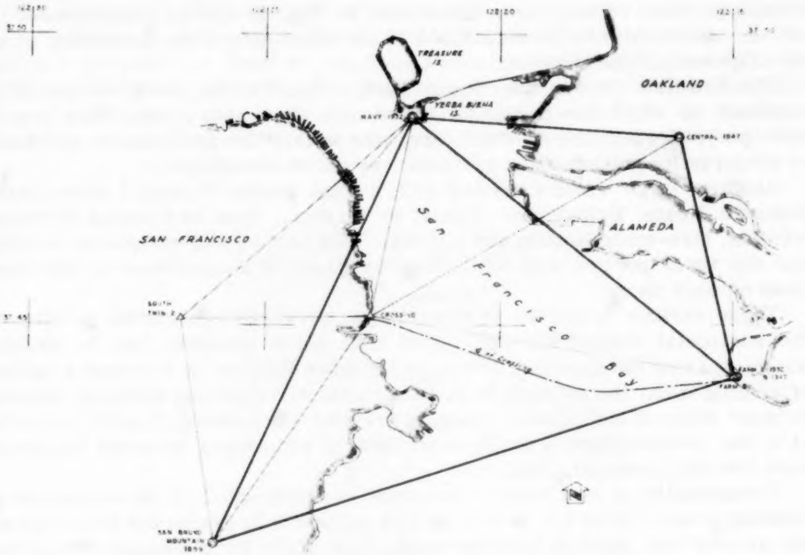

FIG. 3.—TRIANGULATION NET FOR SOUTHERN CROSSING OF SAN FRANCISCO BAY

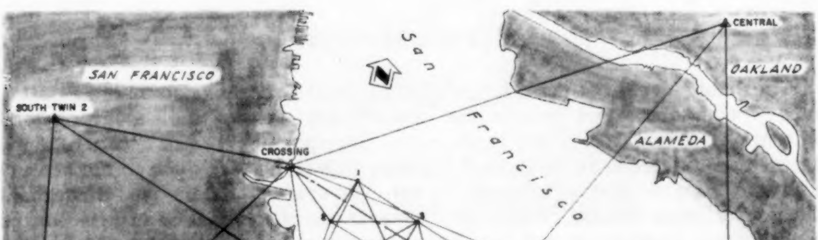

FIG. 4.—CONSTRUCTION CONTROL FOR SOUTHERN CROSSING OF SAN FRANCISCO BAY

For construction control, one of the more distant stations in the primary net will be abandoned and a new land station established in the vicinity of the crossing. This revised net, illustrated in Fig. 4, will be supplemented by eleven control towers located 2,000 ft on either side of the centerline along the alignment of the crossing.

The Sunshine Skyway over lower Tampa Bay, Florida, completed in 1955, required an elaborate triangulation network for construction. This is a 15 mile project, 12 miles of which are over water. The preliminary and final, or construction triangulation systems, are shown elsewhere.[1]

Another large water crossing with a total length of about 5 miles is the Hampton Roads Bridge and Tunnel in Virginia. This is a series of bridge trestles, man-made islands and a tunnel. This project was completed in 1958, and the water portion was controlled[1] by means of a quadrilateral with base lines on each shore.

These various examples of triangulation networks indicate the solution to the horizontal control for the typical long water crossing. For the shorter water crossing the approach is similar but much simpler. It becomes a matter of scaling down the network to suit the particular crossing distance, bearing in mind some of the general concepts previously examined. For the crossing of a few thousand feet, a single quadrilateral with two or even one measured base line will generally suffice.

Occasionally, a new bridge structure is constructed in the vicinity of an existing bridge. In such a case, it may be advisable to design the triangulation net around this existing bridge, using it as a site for a measured base line and also as ameans for spotting pier locations during construction. The net under such circumstances will generally be a quadrilateral or series of them as before with a base line along one side, measured along the existing bridge, and possibly supplemented by another base line on shore.

CONTROL STATIONS

The basic design of the triangulation net will be influenced by the existing survey controls found during the preliminary survey, the topography and bottom characteristics of the location and the type of bridge structure. Having decided on the net to be used, first concern should then be given to the control points or stations that will define the net. The system will be used throughout the construction period of probably several years and will be exposed to the construction operations and to adverse weather, hence care should be exercised in selecting the location and providing protection for the control stations. On land, the main stations should be established on vantage points remote from the construction area and visible from all possible directions. They should also be in dry areas, particularly when the stations determine the ends of base lines.

These land control stations should be promptly and substantially barricaded. Then, as a further precaution they should be carefully referenced. The precision for the control system will generally be better than first-order, and the referencing of control stations must be carried out with precise methods to yield comparable accuracy.

With the land station properly established, protected and referenced, it can then be made ready for use both to occupy with a theodolite and to sight on from other stations in the net. Just what this preparation will entail will de-

pend on a number of factors such as the height of the station above the surrounding land and water, the type of foliage, whether it would be more feasible to clear the foliage or sight over it, and the general climate as related to the heat waves and refraction effects on an instrument line-of-sight. For the smaller projects, or when the stations are on relatively high ground free of growth and offering good visibility, there may be no problem involved, in which case the theodolite can be set up directly over the point when the station is occupied; or a simple target may be supported directly over the point with

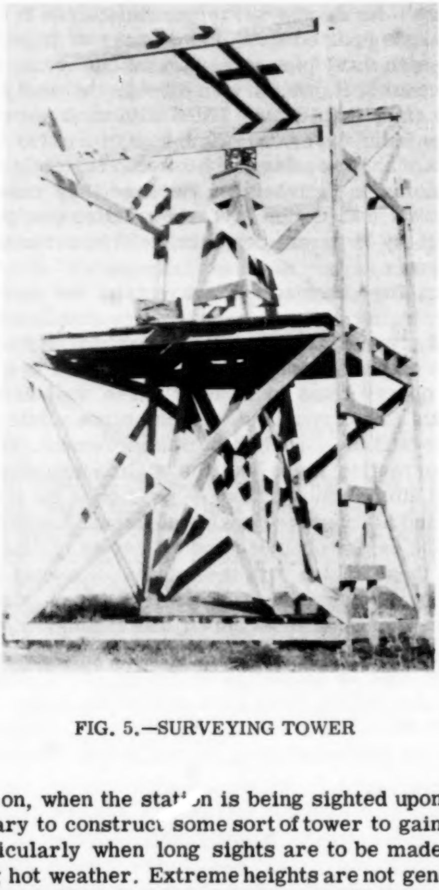

FIG. 5.—SURVEYING TOWER

guy wires and so on, when the station is being sighted upon. Often, however, it becomes necessary to construct some sort of tower to gain elevation for the line of sight, particularly when long sights are to be made over wooded and water areas during hot weather. Extreme heights are not generally necessary. Often a tower 15 ft to 25 ft high is adequate. An effective tower of this sort is shown in Fig. 5, in which an independent central support is provided with a trivet for mounting the theodolite, and an exterior platform free of instrument tower is provided to support the observers. A target for sighting can easily be mounted on such a tower.

Control stations in shoal water areas, with depths not greater than 4 ft or 5 ft can generally be treated as land stations. Monuments should be placed or pipe driven and, perhaps, reinforced with concrete. The referencing should be done in the same manner as for the land stations. Referenced hubs can be placed in a few feet of water and accurate underwater or abovewater measurements made. Any such unusual circumstances as measuring under water should obviously be recorded as part of the reference description in addition to the tape used, pull, support, temperature of water, and so on.

The control stations in deep water, often required for the long crossings of 3 miles to 4 miles or greater, present a much more difficult problem. Substantial towers are often necessary to resist wave and ice action. For a typical tower suitable for depths up to approximately 45 ft or 50 ft, the instrument and target are supported by an inner tower or tripod made up of a prefabricated frame with three pipe piles located on the bay bottom and spudded in place with three steel H piles driven through the pipe piles. The voids between the H piles and pipe piles are filled with sand and gravel. Outside, and independent of the inner tower, is a timber platform for the observers, supported on clusters of timber piles. Such towers are costly and present sizable construction problems in themselves. Because they must be available at an early stage to enable completion of a triangulation net, it is often necessary to construct them by separate contract well in advance of the first bridge construction contracts.

The referencing for checking and recovery of the water control station is not as simple a matter as on land. Towers of this sort should always be checked after bad storms or suspected impact to verify that there has been no movement. One method for referencing such a point is to occupy the station and make angle observations to at least three well-defined and permanent existing landmarks. Check readings on these same points as a later date will indicate movement and the direction of this movement. The point can then be moved back or correction made and the station henceforth occupied eccentrically. A second similar method can be used once the triangulation network has been closed and all observed angles adjusted. A control station or tower suspected of moving can be occupied and all angles to other towers observed. A comparison of these angles with the original observed and adjusted angles should indicate any displacement of the survey point. Movements of a small fraction of an inch can be detected in this manner with the aid of modern direction theodolites.

Another method that is helpful in detecting tower displacement involves setting a point on the railing along each side of the tower and, with the plumb bob, establish corresponding points on the platform. Subsequent displacement of the tower will be evidenced by a change in the vertical alignment of corresponding points.

BASE LINES

The base line for the bridge control network is generally a measured line as contrasted to some known distance between fixed stations that can be used as an "air base." As determined previously[4] the use of existing USC & GS control points can save effort and provide sufficient accuracy during the preliminary phase. However, better than first-order accuracy will be required

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[4] "Highway and Bridge Surveys: Preliminary Surveys," Committee on Engrg. Surveying, Vol. 84, SU 2, Proceedings Paper No. 1697, ASCE, July, 1958.

for the long water crossing and measured base lines will probably be the only answer. Of course, it will be prudent to include existing coordinated stations into the system as a check or to supplement measured base lines, but such an arrangement should be well thought out and the base line from existing control used only on a "weighted basis."

The measured base lines for the long water crossing, generally two in number, must possess first-order accuracy or better to be effective and enable pier locations to a small fraction of an inch. This means that the probable error of a base line must not exceed 1 in 1,000,000. If this degree of precision is to be attained, the base line should be straight and sufficiently long to provide the required strength of figure. Where topography demands that base lines be composed of several tangents, the number of tangents should be kept to a minimum. Under this condition, the terminal stations of the base line must be intervisible so that an angle closure of the polygon can be effected. It is also requisite that the horizontal deviation of all the tangents to the projected base line be less than 20°, and where possible that this maximum be reduced to 12°. Vertical slopes as great as 10% are acceptable and in all cases the difference in elevation between the ends of the tape should be determined accurately. The procedures and equipment necessary to obtain this precision are well covered elsewhere.[2,3]

Triangulation, when used for horizontal control of bridge projects, usually involves considerable distance between stations and sometimes large differences in elevation. For this reason, it is necessary that all horizontal distances be reduced to the same vertical control datum if they are to be comparable. In most cases, this datum is sea level of 1929, as established by the USC and GS.

ANGLE OBSERVATION

Once the control stations have been established and the necessary towers and targets erected, angle observations should begin. This operation should be initiated by furnishing the observation teams with descriptions and a map showing the location and name of each of the control stations. The order in which the various stations are to be occupied should be planned and coordinated so that maximum efficiency can be attained between the field and the office staff assigned the task of reducing the field data and adjusting the net.

Generally, angle observations are made at night in order to take advantage of lighted targets and conditions of minimum refraction. However, under favorable weather conditions, satisfactory observations can be made during daylight hours by using suitable targets.

The choice of a method of observation is determined by the type of theodolite to be used. With the repeating theodolite, it is necessary to measure the quadrilateral angles between successive stations and the angles needed to close the horizon. On completing the observations, several field checks should be made to determine if the observations comply with specifications.

The operation of observing angles usually requires that the observed stations be manned continuously for setting and maintaining the target. It is, therefore, essential to provide a means of rapid communication between the stations. For this purpose, two-way portable radios are satisfactory.

Generally, for a bridge triangulation net, the field procedures and specifications for first-order angle observations, as has been previously[2,3] set forth, must be equalled or exceeded. Under certain conditions, some variations

from these standards are necessary. Terrain and the points to be located may, for example, require that the strength of figure be impaired. However, a thorough analysis of the situation combined with the exercise of proper techniques and scrupulous care in all operations may offset unavoidable deficiencies.

OFFICE COMPUTATIONS AND TRIANGULATION ADJUSTMENT

The office computations required for the construction survey vary from the elementary mathematical procedures necessary to determine an average distance to the solution of the complex systems of equations required for the least squares adjustment.

Office computations, generally, fall into one of four categories. They are the reduction of field data, the determination of the most probable value, the determination of position and the computation of field stake-out data. Some specific examples of each category follow:

1. The reduction of field data includes; (a) correcting measured distances for temperatures, sag, index error, and slope; (b) computation of elevations and differences in elevation from level notes; and (c) computation of angles from notes on observation with the direction or repeating theodolite.
2. The determination of the most probable value involves; (a) weighting and meaning of measurements to determine the most probable value; (b) computation of the probable error in the value so determined; (c) computing and adjusting the error of closure in traverses; and (d) computations required for adjusting triangulation nets.
3. Determination of position comprises; (a) computation of grid coordinates; (b) computation of geodetic position; and (c) computation of astronomic and grid azimuths and bearings.
4. Computation of field stake-out data includes; (a) computation of data for locating points by direction ties, distance ties, or a combination of the two; (b) computation of stake-out data for horizontal and vertical curves; and (c) design of graphs and charts for locating test borings, caissons, suspension cables, and other special purposes.

Many of these operations involve routine computations and are explained in detail in most surveying tests. However, those involved in triangulation adjustment and the determination of grid coordinates and bearings are specialized, somewhat more complex, and require a rather comprehensive treatment.

Triangulation adjustment is basically the application of the theory of least squares to determine the most probable position of the stations in a triangulation net. The exact application of the theory of least squares to triangulation adjustment gives the most correct solution, but also involves a considerable amount of computations. Efforts to eliminate some of the computations from the exact solution have brought about several approximate methods that are satisfactory for adjusting local nets comprising a few stations.

LEAST SQUARES ADJUSTMENT

Although a considerable amount of computations are required for a least squares adjustment of a triangulation net, the use of this method is virtually mandatory for the satisfactory adjustment of the large nets necessary for the

major crossings. A complete example of an adjustment by this method will be included in the final report. A comprehensive treatment of least squares adjustment and the computations for grid coordinates, bearings and other allied data, is available.[3,5]

A method applicable to the adjustment of local triangulation nets, that eliminates the necessity of geodetic computations, involves the application of a correction for spherical excess to the observed or geodetic angles to obtain the corresponding grid angles on the state coordinate system. These grid angles are then adjusted by the usual least squares procedures.

Two other methods of adjustment that are valuable in certain cases are the "Variation of Plane Coordinates" and the "Variation of Geodetic Coordinates."

The variation of plane coordinates method is useful for second-order triangulation adjustment. A publication explaining this method is available.[6]

The variation of geodetic coordinates method is explained in detail in another well documented publication.[7] This method is satisfactory for the location of a few new points relative to a number of fixed stations.

Following the completion of adjustment to the triangulation net, there are still other computations to be completed. Particularly important are the geodetic position and grid coordinate computations. Many states now have or soon will adopt state coordinate systems.[8] Public works such as highways and highway bridges are now commonly being located by these systems. State coordinate systems are developed upon geodetic values, but the use of independent nets is not precluded. The bridge net, however, can be correlated to the larger net.

When a bridge or other crossing is located on a state coordinate system it involves certain important considerations as far as surveys are concerned. It means, first of all, that the theoretical locations of the bridge-heads by the preliminary survey form the basis of coordinates for the entire project. This points out the need for an early establishment of a triangulation system to determine the length of the project and relate the location to the coordinate system. For such projects, it is particularly helpful if one or more previously coordinated stations can be used in the net.

Another consideration involved in locating a crossing on a coordinate system is the constant use of the grid factors. When geodetically computed distances are to be converted into grid distances or vice versa, the grid factors are involved. This factor is the ratio between a given geodetic, or sea level, distance and the distance on the grid between two corresponding points. The values of grid factors, that vary with latitude, together with other data for geodetic to grid computations are given in the plane coordinate projection tables for the various states, published by the USC and GS. Similarly, because a standard grid is a theoretical plane representing a certain portion of the earth's surface, azimuths and angles thereon likewise will not conform to their geodetic values. These differences must be taken into account in all computations involving these systems.

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5 "Triangulation Computation and Adjustment," USC and GS Manual.

6 "Manual of Plane Coordinate Computation," USC and GS Special Publication, No. 193.

7 "Applications of the Theory of Least Squares to the Adjustment of Triangulation," USC and GS Special Publication No. 28.

8 "Highway and Bridge Surveys: State Plan Coordinator," Committee on Highway and Bridge Surveys of the Surveying and Mapping Div., Vol. 83, SU 1, Proceedings Paper No. 1306, ASCE July, 1957.

It should also be noted that when the mean ground elevation is considerably above or below the mean sea level datum, it will be necessary to further correct the measured distances for the difference in elevation. The correction is called "reduction to sea level."

SPECIAL TECHNIQUES IN TRIANGULATION COMPUTATION

Some rather specialized techniques have been found to be extremely helpful in bridge triangulation computations. These systems are significant mainly in that they speed up the project by providing rapid solutions to problems that may be repeated scores or even hundreds of times. Two such techniques will be cited. A third technique is a means of attaining higher mathematical precision. The first technique is used for computing the spherical excesses of triangles, that must be computed in advance of the adjustment because they are used to figure the triangle closures, some of which appear in the angle equations.

The spherical excess of a triangle on the earth's surface is the amount by which the sum of its angles exceeds 180°, and is proportional to the area. (The approximate spherical excess in seconds equals 1 sec for every 75 sq miles in area). It is normally computed by the formula

$$e = \left(\frac{1}{2}\ a\ b\ \sin\ C\right) m \qquad (1)$$

in which m is a numerical factor depending on the mean latitude and units of distance. The USC and GS has presented[2] values of log m for the various latitudes and based on distances in meters. The problem of computing by this formula is quite long by the ordinary method when the known quantities are the coordinates of the three points involved.

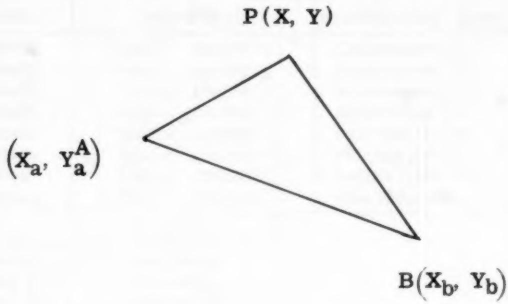
The basic formula process is replaced by four multiplications in the special system. Preparing such a system for a particular project consists of deriving or computing a pair of multipliers for each pair of triangulation stations used in the location work, such that one spherical excess may be computed as the algebraic sum of four products. Each of the four is the product of one of these special multipliers and a grid coordinate.

Once the net for the bridge location has been established, these multipliers can be computed. It is wise to do this in advance of the construction work so that any spherical excess may be computed quickly. A pair of multipliers is computed from the coordinates of the two net stations involved, the value of m and a correction factor for the grid coordinates being in feet. The simplified computation may be expressed as

$$e = M_x\left(X - X_a\right) + M_y\left(Y - Y_a\right) \qquad (2)$$

in which M_x and M_y are the predetermined multipliers, X and Y the design coordinates of the point being located, and X_a and Y_a those of one of the two

stations. The figure below shows a sample triangle. Points A and B are the fixed stations and P the point to be determined with coordinates as indicated:



The following equations may be used to compute the multipliers;

$$M_x = m' \left(Y_a - Y_b\right) \quad \text{. . . . .} \quad (3)$$

and

$$M_y = m' \left(X_b - X_a\right) \quad \text{. . . . .} \quad (4)$$

in which m' is the m from the table divided by 10.763867, the number of square feet in one square meter. For most projects, the range of latitudes is sufficiently small that a single value of m, and therefore m', is satisfactory for all triangles.

It should be noted that, in triangulation for the exact location of a temporary point, the approximate coordinates of the point may be used for the spherical excess computations. If the temporary point is set approximately at some theoretical location, the design coordinates may normally be used. If the point is known to be some distance from the theoretical one, its coordinates may be estimated with sufficient accuracy. In rare cases for very precise work, a recomputation may be necessary after the first adjustment.

The second useful time-saver is a special set of tables (Table 1). Compiled for each job, it is excerpted from various sources to eliminate the time it takes to refer repeatedly to those sources in the computing of geodetic positions and grid coordinates. The USC and GS has presented several special publications[9,10,11] that furnish much of the data for such a table. In few cases will the region involved in a bridge triangulation net cover a range of more than 5 or 10 min in latitude or longitude, so that the abbreviated table is usually feasible. Table 1 gives the ten-place natural sines for the applicable range of the grid mapping angle, θ, and also the values of $1 - \cos\theta$ to twelve decimal places. The values of $1 - \cos\theta$ were developed to reduce the number of steps in the grid coordinate computation. Because $1 - \cos\theta = 2\sin^2\theta/2$, a single operation using $1 - \cos\theta$ replaces three steps in the regular process. By having all of these data readily available on one sheet, and using the shorter process, time is saved on each position determination.

Another special technique of value in triangulation computations is a means of obtaining accurately the logarithmic sines of small angles. In normal

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[9] United States Coast and Geodetic Survey Special Publication No. 241.
[10] United States Coast and Geodetic Survey Special Publication No. 246.
[11] United States Coast and Geodetic Survey Special Publication No. 253.

TABLE 1.—ABBREVIATED TABLE FOR DETERMINING

LAT	MER. ARC (meters)	DIFF/SEC	H VALUE
37° 51'	4,190,643.363	30.830 933	.0409 02930
52	4,192,493.219	30.831 033	.0409 12141
53	4,194,343.081	30.831 117	.0409 21359
54	4,196,192.948	30.831 200	.0409 30585
55	4,198,042.820	30.831 300	.0409 39818
56	4,199,892.698	30.831 383	.0409 49059
57	4,201,742.581	30.831 467	.0409 58308
58	4,203,592.469	30.831 550	.0409 67564
		y' for V	V
		4,190,000	6.082 6318
		4,200,000	6.102 3668
		4,210,000	6.122 1497
LAT	R (feet)	Y' (feet)	TAB/DIFF
37° 51'	27,021,453.86	491,538.18	101.14417
52	27,015,385.21	497,606.83	101.14450
53	27,009,316.54	503,675.50	101.14483
54	27,003,247.85	509,744.19	101.14533
55	26,997,179.13	515,812.91	101.14567
56	26,991,110.39	521,881.65	101.14600
57	26,985,041.63	527,950.41	101.14633
58	26,978,972.85	534,019.19	101.14683
θ	SIN θ	TAB/DIFF	1-COS .0002
1° 09' 00 min.	.020 0699 388	484 716	01421 505
10	.020 1184 104	484 715	02395 700
20	.020 1668 819	484 715	03372 244
30	.020 2153 534	484 714	04351 135
40	.020 2638 248	484 714	05332 381
50	.020 3122 962	484 713	06315 972
1° 10' 00 min.	.020 3607 675	484 713	07301 914
10	.020 4092 388	484 713	08290 210
20	.020 4577 101	484 712	09280 852
30	.020 5061 813	484 711	10273 844
40	.020 5546 524	484 711	11269 186
50	.020 6031 235	484 711	12266 879
1° 11' 00 min.	.020 6515 946	484 710	13266 922
10	.020 7000 656	484 709	14269 315
20	.020 7485 365	484 710	15274 054
30	.020 7970 075	484 708	16281 147
40	.020 8454 783	484 708	17290 591
50	.020 8939 491	484 708	18302 385
1° 12' 00 min.	.020 9424 199	484 707	19316 524
10	.020 9908 906	484 707	20333 014
20	.021 0393 613	484 706	21351 859
30	.021 0878 319	484 705	22373 049
40	.021 1363 024	484 705	23396 594
50	.021 1847 729	484 705	24422 484

GEODETIC POSITIONS & LAMBERT COORDINATES

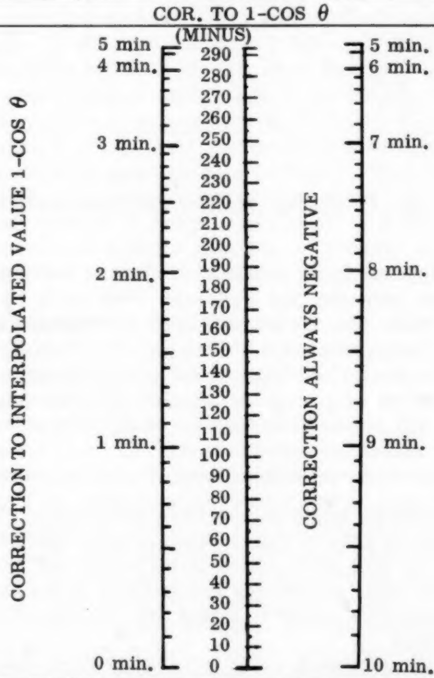
D/SEC	SIN ϕ		D/SEC	f (D/S-5.233)	
153.52	.613	59636	382.78	8.207	5748
153.63	.613	82603	382.68	8.207	5434
153.77	.614	05564	382.60	8.207	5120
153.88	.614	28520	382.52	8.207	4806
154.02	.614	51471	382.43	8.207	4492
154.15	.614	74417	382.33	8.207	4478
154.27	.614	97357	382.27	8.207	3864
154.40	.615	20293	382.17	8.207	3550

Δ V	
197	350
197	829

LONG.		θ ANGLE			
122°	22'	-1°	08'	34.199	327 min.
	23	-1°	09	10.933	249
	24	-1°	09	47.667	172
	25	-1°	10	24.401	094
	26	-1°	11	01.135	017
	27	-1°	11	37.868	940
	28	-1°	12	14.602	862
	29	-1°	12	51.336	785

TAB/DIFF	
974	195
976	544
978	891
981	246
983	591
985	942
988	296
990	642
992	992
995	342
997	693
1000	043
1002	393
1004	739
1007	093
1009	444
1011	794
1014	139
1016	490
1018	845
1021	190
1023	545
1025	890
1028	241

COR. TO 1-COS θ



triangulation very small angles are avoided, and ideally, none smaller than 30° are included. However, in most bridge location work there must be some small angles, due to the fact that some of the points to be located are likely to lie near certain lines of the net.

The logarithmic sines are used first in development of the side equations, and again, after the adjustment is made, in the computation of distances on the triangle sheets. The difficulties involved are quite serious in which small angles are unavoidable and high precision must still be maintained. The source of difficulty is, of course, the rapid change in normal tabular differences for the small angles.

The tables used for this work are based on interpolation intervals of 10 sec. For small angles the sines themselves increase almost linearly, but the changes are so rapid that the deviation for the log sin curve is quite notable, even for these intervals.

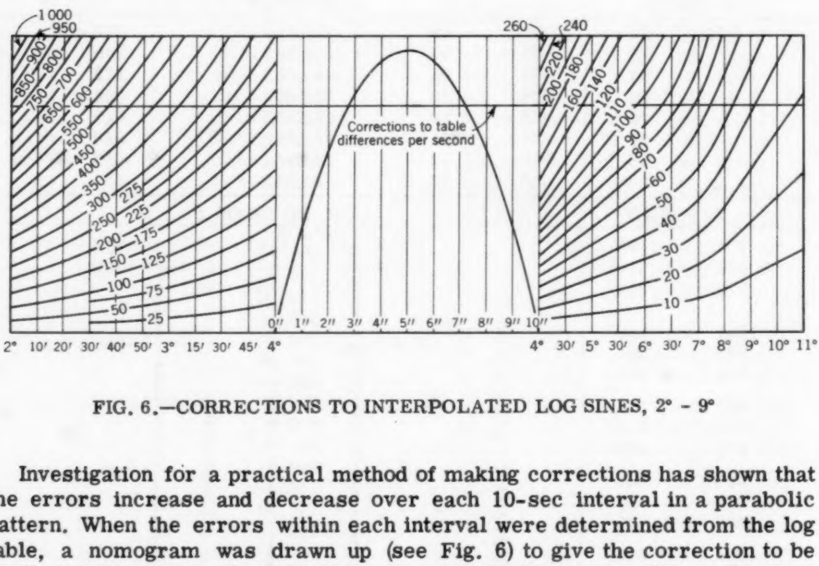


FIG. 6.—CORRECTIONS TO INTERPOLATED LOG SINES, 2° - 9°

Investigation for a practical method of making corrections has shown that the errors increase and decrease over each 10-sec interval in a parabolic pattern. When the errors within each interval were determined from the log table, a nomogram was drawn up (see Fig. 6) to give the correction to be applied to the value obtained from the straight-line interpolation. Because the nomogram is purely an adjunct to a ten-place log sin table, it would be useful in all cases when high accuracy is desired in the use of ten-place log sines for angles between 2° and 11°.

Fig. 6 is to be used as follows: To find corrections to Log Sines;

1. Beginning below the arched center curve at the number of seconds, the given angle is greater than an even 10 sec; follow it vertically up to the curve.
2. From this point follow horizontally left or right to the vertical line up from the number of degrees and minutes in the angle.
3. Interpolate along this line between two adjacent curves for the amount of correction.
4. This correction is in units of the tenth decimal place and always to be added to the value of the log sine obtained by ordinary interpolation.

To find corrections to differences for side equations;

1. Follow up the vertical line from the number of degrees and minutes of the angle to the horizontal line marked "corrections to table differences per second," and interpolate between the adjacent curves to this intersection.
2. Multiply this value by the number of seconds the given angle is above or below an even 5 sec, 15 sec, 25 sec, and so on, to compute the toal correction.
3. This correction is in units of the tenth decimal place, and is to be added to the given table difference if the angle is below the even 5 sec, subtracted if above the 5 sec. It should be noted that the corrected differences obtained by the procedure are not to be used for interpolating but only for side equation coefficients.

It should not be inferred that the use of this device eliminates the harmful effects of small angles on the strength of a figure. It does, however, make possible a good closure, trigonometrically, when small angles must be included in some triangles.

SURVEY CONTROL PLATFORMS

In most cases, pier construction is controlled through the use of control platforms in addition to the basic control towers that define the triangulation net. Because the method of bridge construction dictates the survey procedures it also determines the platform requirements. Bell pier construction, involving no cofferdams, forms or caissons above water during the early stages, has a greater need for platforms close by than do the other methods. Where the cofferdam method is used, construction control points may be located on the cofferdam from the known triangulation monuments or towers.

For the Richmond-San Rafael Bridge bell piers, the platforms were set 150 ft south of the bridge centerline and opposite each pier. This facilitated quick location and checking of underwater construction of the bell piers. At each pier, construction was controlled from a minimum of three adjacent platforms, using the intersection method for aligning the targets on the submerged units. The preliminary location of the platforms was also done by the intersection method.

With instruments set up on three or more of the land triangulation stations, precomputed angles were turned to give the erection crew at the platform site the proper location. Once the platform was set up, the exact theoretical point for control of piers was located on it by triangulation.

Location of these points for control of underwater construction need not be, generally, as exacting as the final locations for pier details.

In Fig. 4, a series of stable semi-permanent platforms were designed for location of a long crossing. This arrangement of only eleven platforms was planned for control of a combination low-level bridge and underwater tube approximately 7-½ miles in length. The individual platforms are to be located approximately 2,000 ft from centerline and each is to be used for a number of piers. For this design all eleven platform control points are located by a single triangulation problem. The net in this problem involves five previously established land stations, one new land station, and the eleven platforms. This system provides a coordinated set of bridge location control points based on

the net established in the preliminary stage. The preliminary net, shown in Fig. 3, served not only as the basis for final construction but also for the design.

LOCATION OF PIERS

The location of bridge piers involves the most precise control required in the construction survey program. It is a common erection procedure to assemble an entire bridge span on a barge at dockside, float it into position and lower it onto the pier. Obviously, an operation of this magnitude presumes the precise location of piers and anchor bolts.

Pier location is generally effected in two stages. The initial stage provides the control for locating the underwater section of the pier and the final stage establishes the control for locating the above-water portion including anchor bolts, shoes, columns, or other detail as required.

The survey procedures required for pier location will be affected by such factors as the location of the pier relative to land, the proximity and quality of existing control, the type of pier, and the method of construction. Generally, the underwater construction can be controlled by the method of intersection from stations in the net or from control set on survey platforms in the immediate vicinity of the pier. The precise control for the anchor bolts and the other above-water components is usually established by triangulation from first-order stations or possibly by angle and distance measurements if control towers are in close proximity of the pier. In the following sections these factors will be discussed in detail relative to specific types of piers.

DRY LAND PIERS

Included in most major bridge plans are the extensive approaches to the water crossing. The horizontal control for the "dry land" approach piers may offer special problems, but the solutions will generally be straightforward. The land approach alignment will be a matter of establishing traverses connected with the triangulation network for the water crossing. Location control for the piers along the approaches can generally be handled by establishing offset lines parallel to the alignment traverses, then locating trasverse centerlines of the piers on the offset lines from which the piers can be spotted as needed by turning 90° from the offset lines. Many bridge approaches are on horizontal curves that will complicate the problem, but the approach can be similar. Offset lines parallel to the curve tangents may be established, and points located where the transverse pier centerlines intersect the offset lines. It then becomes a simple matter of turning angles and measuring tangent offset distances to locate the piers on the curve. This method will have many variations depending on the terrain and existing topography.

Frequently, the process of locating piers on curved alignment can be greatly facilitated by placing the alignment and the control traverse on a coordinate system. The locating procedure can then be reduced to computing and making angle and distance ties to the piers from convenient traverse stations. Approaches occurring in congested metropolitan areas will greatly restrict the surveying necessary for the control; however, the solution will generally be

a matter of trigonometry with the use of various offsets and radial lines when curves are involved.

COFFERDAM PIERS

The survey control required for this type of pier construction is divided into three stages. The initial stage provides the control for locating the cofferdam. The second stage establishes the working points on the cofferdam for directing the pier construction and the final stage establishes the precise control on the pier.

The method of intersection utilizing three or more stations is an effective procedure for locating markers around the dredging area, anchors, guide piles, falsework piles, and other components required for the construction of the cofferdam. When the intersection method is supplemented with the previously prepared "locator charts" described in the section dealing with bell piers, the combination is an exceedingly fast and reliable method for directing the various components into the proper position.

For some conditions, cofferdam construction requires locating a bracing frame where the cofferdam is to be erected. This can be accomplished by placing vertical rods at three corners of the frame and aligning them by intersection from the control station. The vertical alignment can be controlled by graduating the rods so that a level reading on all the rods will be the same when the frame is vertical. In tidal waters, or where strong currents prevail, frequent checks should be made during the construction of the cofferdam to assure its proper location.

Accurate working points and temporary bench marks for locating bearing piles and controlling pier construction may be set on the upper ring of wales of the completed cofferdam. These points should be checked when the cofferdam is unwatered and periodically thereafter. Bench marks should also be placed in the tremie seal. Soundings for determining the elevation of the tremie seal can be conducted from control set on the top ring of wales and struts.

When the pier reaches the proper elevation, the permanent centerlines should be established in the design location from the working points on the cofferdam. These centerlines should be checked from the control stations prior to locating pedestals, anchor bolts, base plates, and other similar details. Permanent bench marks should be set at the four corners of the pier and at the center to facilitate checking future settlement of the pier.

CAISSON PIERS

Survey control for the caisson pier is also effected in three stages. The first stage locates the facilities at the pier site prior to floating the caisson into position. Stage two directs the sinking of the caisson and the final stage establishes the precise control on the pier.

As in the construction of the cofferdam pier, the method of intersection supplemented by the "locator charts" is equally applicable to locating marker bouys, caisson anchors, guide piles, temporary piles for the construction docks, and the other facilities required for caisson pier construction.

Prior to locating the caissons and piers in the field, the use of the "detectograph" provides an excellent method for detecting large errors in the precomputed angles between the triangulation stations and the centerline of the pier.

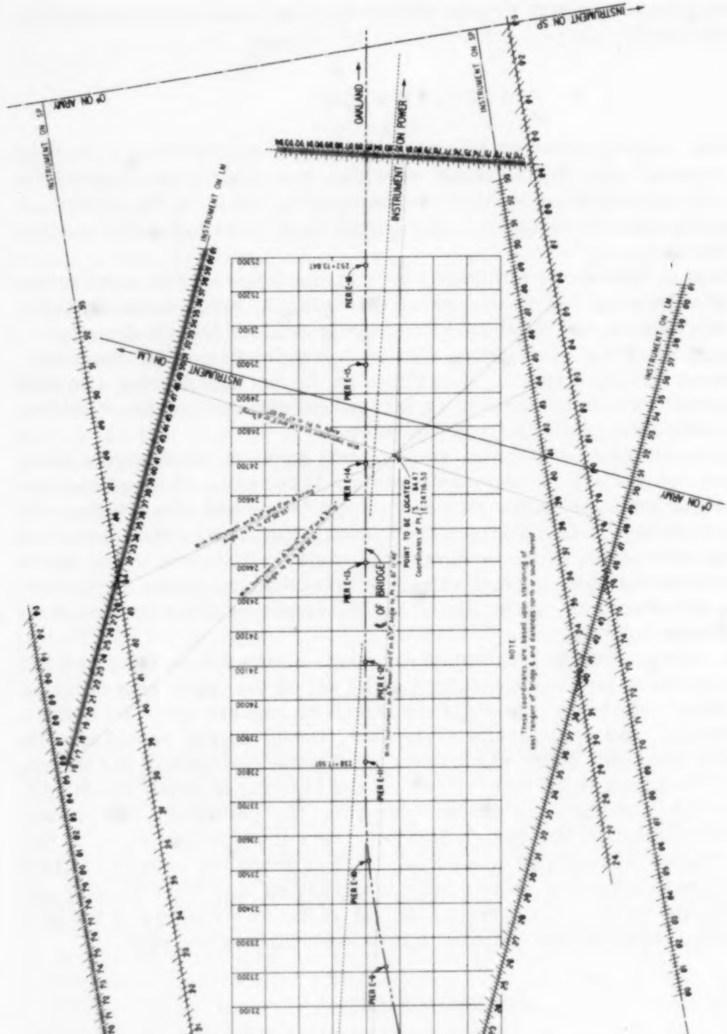


FIG. 7.—SAMPLE DETECTOGRAPH

The detectograph, Fig. 7, is merely a large scale map of an area on which angle graduations define the direction of observations taken from triangulation stations. The intersection of these directional lines graphically defines the points observed.

After the caisson has been anchored at the pier site between previously located guide piles or working docks, or a combination of them, its location should be checked by observations from the control stations or working points previously established on the working docks. These observations should continue as the caisson progresses downward through the water. The anchor lines should gradually be adjusted (if anchors are used to restrain the caisson) in order to bring the caisson to its design location just prior to landing it on the bottom.

The position of the cutting edge can be determined by accurately locating two corners of the caisson and making corrections for the list. One fast and simple method for accomplishing this is the use of platforms at the two corners to be observed. A coordinate system is marked on these platforms in feet and tenths, using the corner of the caisson as the center of coordinates. The design location of the cutting edge may be determined by turning precomputed angles at the control station or by sighting on previously established foresights that have been checked by angle repetition. A range pole at each platform is then aligned simultaneously by observations from three control stations. The position of the range pole will indicate the design position of the corner, and the divergence of the actual corner can be read directly from the coordinate system. The list may be determined by setting up a transit on the caisson with its vertical axis normal to the plans of the cutting edge. The angle between the cutting edge and the horizontal is read from the vertical arc when the telescope is leveled. This angle should be checked by readings taken from a plumb line suspended inside the caisson.

The depth to the cutting edge of the caisson must be available throughout the sinking operation. One positive method makes use of sections of pipe milled to equal lengths and fitted with couplings for connecting the sections. While on dry dock the first sections of pipe are set at each corner of the cutting edge a measured distance from the bottom so that a plane parallel to the cutting edge is defined by the top of the pipes. As the caisson is sunk, additional sections are added as required and a record kept of each. With this procedure, the depth to the bottom of the cutting edge is known at all times.

When a caisson is anchored in tidal waters it moves in a rhythmic orbit depending on the magnitude and direction of the currents. This movement should be observed continuously for several days prior to landing the caisson on the bottom. The results of these observations should be charted to show the effect of the tidal currents on the position of caisson. This data can be helpful later to predict caisson behavior during the sinking and landing operations.

In order to land the caisson in its design location, directions for the final adjustment of the anchor lines must be given as the caisson sinks the last few feet as it nears the bottom. It is therefore necessary that the position of the cutting edge be known at all times during this stage. This will require checking the position of the caisson as frequently, perhaps, as at one minute intervals.

One method of control that can be used to give location with this frequency is the method of intersection, utilizing the locator chart, as previously mentioned. The effective use of this method depends on the advance preparation of the locator chart, the charts giving the corrections for list and all other data required to facilitate the operation.

When the landing operations begin, these previously prepared data come into important use in giving the location of the cutting edge immediately after the observations are received from the control stations. The observations should begin sufficiently in advance of high water slack and continue until the caisson comes to rest on the bottom, after which the corners of the caisson should be checked and the list determined.

The position of the caisson should be checked continually until it has reached its final depth. At that time the final soundings should be taken to the bottom of the dredging wells and subsequently to the top of the seal after it has been placed.

The precise control on the pier for the pedestals, anchor bolts, base plates, and other details should be established from the control stations by triangulation. Bench marks should be established on the pier near each corner and their elevations determined from known bench marks on shore or from bench marks previously set on completed adjacent piers. Where the overwater distance between piers is beyond the realm of differential or reciprocal leveling, the Transbay or River Crossing Method may be used. For first-order leveling, the Transbay leveling should be run at night.

BELL PIERS

The method of underwater pier construction that requires the most specialized survey techniques is that of using precast concrete or prefabricated steel bells. This method presents a rather special problem of control during the early stages when the work is actually being done under water. The basic steps in this construction procedure are as follows:

1. Excavation of soft sediments.
2. Driving timber falsework piles and placing the base grid.
3. Driving steel bearing piles.
4. Placing underwater bells and pouring tremie concrete.
5. Completion of pier shafts, anchor bolts, and so forth, above water by conventional construction methods.

Because the greater part of the construction for this type of pier is done under water by floating equipment, the control problem is made increasingly difficult by the absence of falsework or other stable platforms on which control points can be set, and by the inability to observe directly the component to be located. In order to overcome these difficulties, substantial survey towers should be erected near the pier site and the components placed below the water should be equipped with an aligning tower or target by which their location can be directed from the surface.

For the excavation, three survey operations are necessary; soundings before excavation, control for the excavation operation, and soundings after the excavation. Soundings for determining pay quantities of excavation must be rather exact and carefully located. Electronic sounding gear has generally replaced the old lead-line methods, particularly on large projects. The locating of the sounding boat may be done by measuring the distance and angle from a single survey platform or by simultaneous angles at two nearby fixed stations. Control for the excavations themselves may be accomplished in various ways, such as by setting marker bouys, or by locating the dredging rig in one of the above ways, or by the use of sextants and range charts.

Driving temporary piles on which the basegrid rests, or guides for permanent piles, under water, is the next principal step. Piles for base grids generally need not be located with high accuracy. The grids, on the other hand, must be set within specified limits of the design positions. They must, therefore, be equipped with some means for checking their location under water. Aligning towers for this purpose may be of various sorts. Some device for checking inclination must be incorporated in the tower. The tower's position with respect to the grid must be accurately set and known, so that observation on the top of the tower will indicate the exact position of the grid when it has been lowered onto the piles.

Another valuable aid in locating such underwater units are "locator charts" (Fig. 8): that are prepared in advance. The chart includes the lines of sight, angles to be turned for the true position, the offset in feet for each minute of difference between the theoretical and the observed angle, and a scaled grid

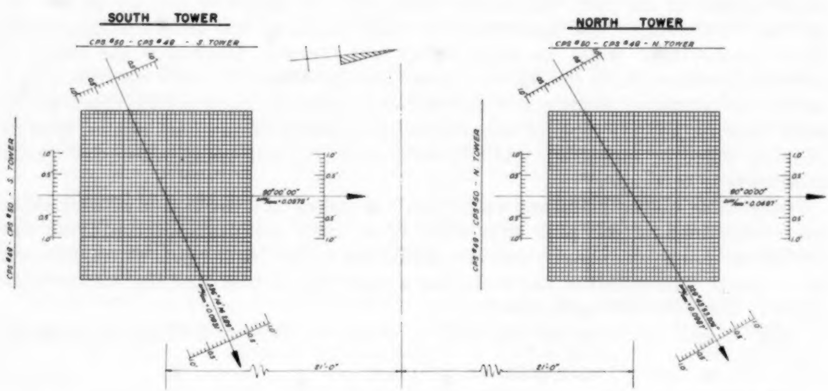


FIG. 8.—EXAMPLE OF LOCATOR CHARTS

for plotting positions. With this prepared information, and the angles to the aligning tower at the instrument stations, the position of the aligning tower can be plotted rapidly and the information relayed to the barge superintendent. The direction and distance for further movement is then readily apparent. This is not only a particularly rapid but a relatively fool-proof technique for directing such activities. This particular type of chart is just one of a number of forms that may be used. Variations of the basic principle may be more suitable under certain circumstances.

The task of driving piles underwater becomes more or less routine once the grids or guides are set. No special control problem is involved in this work. Placing of bells or forms for tremie concrete is controlled in much the same manner as was previously described for the grids. Aligning towers and locator charts again serve to facilitate the operations. When the precast concrete bells are used, the number of sections to be lowered depends upon the depth, but does not require any basic change in method. Frequently, the sections are so interlocked that once the initial section is properly aligned,

control is no longer required. However, in some situations it might be desirable to observe the alignment during the placing of concrete in order to detect any excessive movement that might occur.

PRECISE CONSTRUCTION LAYOUT

The foregoing comments concerning pier locations have been confined, principally, to location of the portion of the pier below the water. Obviously, the difficulties involved in deep underwater construction would make it virtually impossible to maintain extremely close tolerances. This is usually reflected in the tolerances allowed by the specifications for locating underwater components of the pier. The surveying procedures described for the various methods should produce results well within the limits of the specifications.

When construction of the pier has been completed a few feet above the water line, it becomes necessary to establish the centerlines for control of the remaining part of the pier, the anchor bolts, and the shoes or column bases required. Frequently, the specifications will stipulate the limits within which these centerlines must be established and thereby determine the order of precision within which the survey must operate. Where no such specifications exist, the engineer must make this decision after giving due consideration to such factors as the type of superstructure, length of adjacent spans, type of bearing (fixed or movable), and any other condition that would affect the limits of allowable tolerance.

Depending on the accuracy required, the choice of a method of establishing the centerlines of the pier may vary from angle and distance ties from established points on nearby survey platforms to the least squares adjustment of a small triangulation net including a point on the pier and the visible stations in the main triangulation net.

The steps involved in locating a point by this method are as follows:

1. Set an approximate point on the pier.
2. Select the main net stations to be used.
3. Lay out schedule of observations.
4. Prepare forms and complete preliminary computations.
5. Make observations and station adjustments.
6. Make adjustments and compute coordinates.
7. Compute tie and set desired control point.

The approximate point is set as close to the desired location as is practical.

In most cases, further construction at the pier site will be delayed pending the establishment of control on the pier. For this reason, work must be planned so as to get the results as quickly as possible. With stations on opposite sides of, or surrounded by water, much time may be lost in moving observers and instruments. For economy as well as to hasten the completion, good planning is of vital importance. For the same reasons it is wise, if possible, to use more than one instrument in such work because many men and several boats may be involved.

In order to be ready to use the field observations as soon as they become available, and thus minimize construction delay, all possible computations, and so on, should be done in advance.

Certain of the computation forms may be completed up to a point. The direction numbers, stations, spherical excesses for large nets and known

logarithms on the triangle sheets, and certain known data on the geodetic and grid coordinate forms may be entered in advance. In most cases, the figure may be drawn and ready for the observed angles to be entered. When the number of stations is six or more, having the adjustment form drawn up beforehand will mean a saving of time. Because simple quadrilaterals may be used in many cases, a special form for their adjustment is advisable.

Rarely will the approximate point set on the pier prove to be in the theoretical position required. It will usually be necessary to compute the azimuth and distance from the approximate point to the desired control point, based on their coordinates, and with this information establish the control point on the pier.

Several other methods are available for adjusting one point to the fixed stations in a net. These methods require considerably less computation than the least squares adjustment, and give results that are usually satisfactory. Among these methods is the "variation of geodetic coordinates," that has been mentioned in the section dealing with office computation and triangulation adjustment. Another method that should prove useful has been presented elsewhere.[12]

The use of the "three point problem" to determine the position of a point has been generally circumvented because the graphic solution does not produce sufficiently accurate results, and the analytic solution is extremely laborious and susceptible to error. However, the Maryland Bureau of Control Surveys and Maps has presented[13] a method in which the computations involved have been reduced to a simple routine. The report also suggested measures for detecting and guarding against mistakes. This new approach to the solution of the "three point problem" will probably encourage widespread use of this method for determining position.

Experience in using the three-point problem on the Richmond-San Rafael Bridge demonstrated that with precise procedures and extra fixed points a pier point could be located a minimum of 0.02 ft from its most probable position.

It is difficult to define the limits within which geodetic factors such as spherical excess and reduction to sea level do not require consideration for locating piers. For a general rule, if the distances involved are such that no significant change occurs in the length of lines or the size of angles, when corrected for geodetic factors, these factors need not be considered.

Except for relatively short spans, piers are generally located independently of each other. Under these circumstances it is often desirable to measure the distance between adjacent piers in order to verify their locations. Several methods have been used to determine this distance, with excellent results.

One of these methods involves the use of a high quality steel wire in the same manner as the surveyor's tape. Measurements are possible by this means over distances up to about 1,000 ft.

Caution must be exercised that the tension applied is not sufficient to result in a unit stress greater than the yield stress of the wire. In making measurements, the wire should be supported at the ends by functionless pulleys suspended from a tripod (tireless ball-bearing tricycle wheels make excellent

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[12] "Analytics as a Substitute for Triangulation," by the Maryland Bureau of Control Surveys and Maps.

[13] "A New Solution for the Three Point Problem," by the Maryland Bureau of Control Surveys and Maps.

end supports). Uniform tension can be applied by previously computed lead weights suspended over the pulleys or wheels at the ends of the wire.

Another method of inter-pier distance checking is an improvised subtense technique. A short transverse line is laid off at one end of and perpendicular to the line to be measured. This transverse line must have its midpoint coincident with the end of the unknown distance. From the angle subtended by the transverse line and its length, the desired distance is computed.

SPECIAL EQUIPMENT FOR TRIANGULATION

The exceptionally long distances involved in triangulation and the precision required in performing the work precludes the use of much of the equipment normally associated with surveying, and has led to the development of equipment especially suited to the requirements of triangulation. The equipment varies from the small, heavy tripod used to support the tape when measuring base lines, to the elaborate Bilby Towers for supporting the observer and instrument with measuring angles. Much of this equipment has been standardized and is described in USC and GS manuals and in textbooks on first-order surveying.

However, the variety of unusual conditions encountered in triangulation makes it difficult to standardize procedures and equipment to handle each of them. Frequently, the only solution will be found in developing new equipment and methods to solve specific problems. This should be constantly kept in mind so as to avoid becoming encumbered by convention.

SPECIAL EQUIPMENT FOR DISTANCE MEASUREMENT

Recent scientific advances in the fields of physics and electronics have lead to the development of two instruments, the Geodimeter and Tellurometer, that are capable of measuring distances as great as 40 miles with exceptional precision.

The introduction of these instruments has initiated a new method for establishing horizontal control over a large area, particularly in rugged terrain. This method, designated Trilateration, depends on precisely measuring the distances between stations as opposed to measuring the angles and a base line in the usual method of triangulation. Under some conditions it may prove advantageous to combine the use of the Geodimeter or Tellurometer with the use of a theodolite for establishing control. The time required for setting up the instrument and measuring the distance varies from one-half to one hour. The Tellurometer can also be used when atmospheric conditions would prohibit observations with a theodolite.

The magnitude of the inherent error in these instruments is such that the results approach first-order accuracy only when the measured distances approach the maximum range of the instrument. Considering the length of lines normally found in bridge triangulation nets, this characteristic would tend to minimize the importance of these instruments in bridge triangulation except for the long overwater crossing such as the proposed Southern Crossing of San Francisco Bay.

The Geodimeter measures distances by projecting a highly collimated light beam to a distant reflector. The light is reflected back to the instrument

and by special electronic techniques, a phase comparison is made between the light being projected and that being received. Models of different orders of precision are available.

The Tellurometer, which is the newest of the two instruments, is relatively lighter. It utilizes a micro-wave system for distance measurement and can be operated in daylight, dark, fog, mist, or light rain. Although visibility is not necessary, the Tellurometer does require an unobstructed line of sight.

TRAVERSE CONTROL

Applications, Procedures and Specifications.—The traverse system of horizontal control has an important place in bridge surveys. For major bridges it is the main system of control for inshore piers and the highway approach structures. Precision should be the prime consideration in all traverse surveys because such surveys are the means by which the construction of the inshore structures are coordinated to the main overwater crossing. The well known procedures of this type of surveying are refined to a degree not often required for routine highway surveying. Accurately standardized tapes, tension balances, and tape thermometers should be used when first-order results are required. The specifications for the first-order traverse are as follows:

1) Number of azimuth courses between azimuth checks not to exceed 15.
2) For the astronomical azimuth; the probable error of result is 0.5 sec.
3) Azimuth closure at azimuth check points not to exceed 2 sec $\sqrt{N}$ or 1.0 sec per station.
4) Distance measurements accurate within 1-in-35,000.
5) After azimuth adjustment, closing error in position not to exceed 0.66 ft $\sqrt{M}$ or 1-in-25,000.

In these expressions N is the number of stations for carrying azimuth, and M is the distance in miles. The expressions for closing errors, (3) and (5), in traverse surveys are given in two forms. The expression containing the square root is designed for longer lines for which higher proportional accuracy is required. The formula that gives the smaller permissible closure should be used.

Traverses at each end of the overwater structure should be tied into and adjusted to the main triangulation net. An open traverse should never be used to establish control of any importance.

Traverses may also serve various other purposes, such as to make direct measurements coordinating the bridge control to control set by other agencies, or for expanding existing control.

Because the traverse system of control is familiar to all engineers it will not be treated in detail here. The prime difference between the normal application of the traverse and the application of the traverse to bridge construction and control is the time and effort devoted to obtaining accurate results.

CREDIT

This report is one of a series of professional contributions of the Committee on Engineering Surveying:

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SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

THE BUILDER OF THE NEWPORT (RHODE ISLAND) TOWER[a]

From Private Correspondence With Author

EDWARD ADAMS RICHARDSON[1].—The following points have been raised in private correspondence and will be discussed by the author.

1. The lowest placed window, W3, had its bottom edge so placed that the fire-place light might be seen at the harbor shore.

(A) In Fig. 2, the dot and dash line is drawn from direct fire, past the wooden sill to the shore of the harbor, proving that this consideration did govern the window location. This line is used to limit the height of the proposed outer wall parapet.

2. It is recommended that a trench be dug outside of the tower to locate any foundations of exterior construction. As a temporary apse might be added to the east, this direction is suggested.

(A) Mr. Godfrey dug a trench in 1948, from SSE to NNW starting nearly 100 ft outside of the tower and to the south. No traces of "proposed outer wall foundations" were found; hence, the author has assumed the design was made, but this wall was to be a later addition to the structure.

3. It is suggested that a mirror could be used for signalling from the windows.

(A) The author could imagine many methods of signalling, particularly to advise a vessel in the narrows of an error in its course, as by interrupting the west window light. The use of the fixed portions of the tower for signalling could be established, whereas auxiliary means could not, but could be assumed to be available. A mirror might have had some use, although this is doubted.

4. It is suggested that the windows of the first floor were splayed both outward and inward so light might be reflected from the illuminated inner splays to the outer, and whereby the window might be visible outside of the beam of direct fire-light.

(A) This suggestion is made, forgetting that after a vessel has passed through the whole range of direct fire-light, it can still see the window by the wall-light behind. However, near the extreme range rays, when the effective window width approaches zero, there might be some slight benefit from reflection from the splay surfaces. It is not seen that this was part of the design, however.

5. It is suggested that the "hearsay" testimony of Gibbs to Lossing may have been fabricated by Gibbs. Certain statements of Gibbs were in error.

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[a] February, 1960, by Edward Adams Richardson.

[1] Publications Dept., Bethlehem Steel Co., Bethlehem, Pa.

He stated the columns rested on rock. Also, it seems odd that the story of Coddington seeing the tower in 1639 came down to Gibbs, not to the Coddingtons.

(A) If Gibbs did invent the story, it of course has no value. However, certain exceptions made to it are not warranted. Suppose someone did report that the tower was built on clay so rock-like in character that it could be excavated by pick only with difficulty. In the course of time, and lacking technical knowledge of the clay, the story will report a "rock foundation" and still not prove the total error of the tale. Other detail objections to the truth of the story seem to be in this category. In a period when sages are not learned by rote by each younger generation, it is fortunate if one member of a generation remembers the story. It need not be a blood relation. As pointed out in the case of the Whittemore story, it went Whittemore, to Whittemore, to Newton, and to Richardson. Why should it be strange that a Gibbs rather than a Coddington should remember a Coddington story?

6. Erratum: Mr. Godfrey did not class the site as a whole as "undisturbed," merely certain parts of the "construction trench."

(A) This whole question of "distrubed" versus "undisturbed" is in an unsatisfactory state. The erratum more exactly represents the written views of Mr. Godfrey.

7. Erratum: The "construction trench" was not rock-filled, but it was circular and continuous under all columns, and was not only filled radially, but circumferentially only under each column.

(A) It is quite possible that the whole trench was started on the assumption that the tower must be erected on a relatively soft layer of clay and that the trench must be rock-filled throughout, at least on the bottom. However, after going down approximately 4 ft, it was found that a hard clay underlay the soft. The engineer might well have decided that in view of the hardness of the clay, the lack of workers and the difficulty in getting stone, that it would be sufficient to use piers to the hard clay. Due to the hardness of the clay, only a small amount of such material was excavated at each pier site.

8. Objection was raised that construction debris need not be found, provided the area was swept clean and a gravel floor was laid, a few traces of which were found.

(A) This is a reasonable objection.

9. It is suggested that Fig. 3 ignores wooden frames in the first floor windows. These were made of wood 4-in. square.

(A) It will be seen in Fig. 3 that all extreme rays may be determined by the splaying, as such rays do not go near wooden frames. Also, see the attention given to the wooden frame in Fig. 2. (See item 1). When it became evident later that the wooden frames might affect the range of direct firelight at W3 and W3 allowance was made for them.

10. It has been suggested that the tower was built as a "folly" by Governor Arnold, using the antique manner. It is further urged that in colonial times, a fireplace with wall ports instead of chimney was still in use when projecting eaves or other superimposed structure might make a chimney undesirable.

(A) For a man such as Governor Arnold, a "folly" might be merely a "front" for guiding smuggling vessels or privateers to port at night. This assumes, of course, that a man would choose such an expensive manner for creating a front. It also assumes that the tower was built by Arnold. But Arnold, if

he ordered such a signalling system produced, knew about chimneys and should have insisted on one that would assure a fire however the wind might blow. The insistence on a colonial date for the building of the tower rests on such things as a "pistol" flint that might have been from a much earlier tinder box, a piece of pottery of negligible size, a footprint, a tiny piece of glass, and pieces of "clay" pipes. It is essential that these be proved lost by the builders, and also that it be shown that earlier persons could not have left equivalent material. It must be remembered that, if earlier peoples built the tower, it was left virtually undisturbed for nearly 300 yr; little or no change in levels would occur in that period, although much has happened in the shorter time since the colonial period.

11. Through a study of the perspective of tower pictures, it can be shown that the outer wall is not cylindrical, but slightly battered. In the same way, it can be shown that the tower has a slight tilt to the east.

(A) These items are of interest. It should be noted that an eastward tilt of the tower is consistent with partial observations by Mr. Godfrey. He noted that the plaster coating of the foundation stones under the east column had disintegrated. It was in this place that he found the stones loose enough to permit him to slip his arm into the pier up to the elbow. Now prevailing winds, particularly strong winter winds, are from the west or northwest; with a clay foundation, a somewhat greater pressure might be found on the clay to the east than that to the west. Over sufficient time, more water might be taken up by an overcompressed clay at the west than at the east, so, over a long period of time a noticeable tilt might be set up. During this action, loose stone might be slightly displaced at the east pier with consequent plaster disintegration. It is doubted that plaster cracking caused the tilt. It has been suggested that a windmill would add to the pressure and to the effect.

12. It is to be noted that in the paper, the structural analysis is based on a very simple methods and assumes that the outer structure is so proportioned that the column loadings are central.

(A) Instead of limiting the floor beams to what they could carry as a uniform load (no wall support to flooring), suppose the flooring is simply supported at the wall and at the beams and, suppose further, that the load carrying ability of the beams is determined from the resulting distribution of loading span-wise. In this case, the gross floor loading raises to 123.1 psf or a net loading of 109.6 psf. This latter figure is greater than the original figure of 76.0 psf and is closer to the 100 psf live load for public buildings. a simpler assumption might be that each beam took a uniformly distributed load on a 6.15 by 17.48 section of flooring, the rest of the load being wall-supported. In this case, the net uniform loading rises to 98.7 psf. This is probably the simplification used by the tower designer.

These loads lead to increases in the total floor loads. Using 31000 lb per floor, it can be shown that the maximum stress in the columns becomes 74.32 psi, that is very close to the specification standard of 75 psi. This does not include the outer structure. Now, instead of aiming for a purely central loading of the column, suppose the exterior structure is increased to produce reverse bending moments. If the total load from the external roof is 66,000 lb, the extreme fiber stress falls from 74.32 psi to 71.39 psi, though the direct stress rises from 58.13 psi to 64.99 psi.

The above 66,000 lb from the outer structure assumes that the full bending moment of about 317,000 in.-lb is taken by the roof beams. This assumes six-

teen roof beams 21 ft and 6 in. long or a structure diameter of about 66 ft inside of the outer wall. These proportions were used in Fig. 2.
teen roof beams 21 ft and 6 in. long or a structure diameter of about 66 ft inside of the outer wall. These proportions were used in Fig. 2.

It will be seen that the tower of Fig. 2 has been so designed that the columns will not be overstressed under partial loading even though that partial loading lasts indefinitely. Hence, we feel sure the tower was designed to be built in stages. It will be seen also that the design insures full use of the roof beam and roofing material, as this design stresses the beams to 317,000 in.-lb instead of the order of 258,000 in.-lb for the case originally given. The results of this disposition of material in the design is a structure with rather pleasing proportions.

In conclusion, it has been called to the attention of the author that H. B. Holand has written a paper[1] on a member of the Paul Knutson expedition who had the necessary character to give some feeling of face to the "builder of the tower." The "English scientist" is one Nicholas of Lynn, belonging to orders, who could use an astrolabe and magnetic compass, and who was rated as an astronomer. Although not so stated, he could have been trained in church construction and design. He studied magnetic declination from 54° N. on the Labrador coast and throughout the Hudson Bay region. He virtually located the north magnetic pole and he reported his discoveries in a paper, "De Inventione Fortunata," to the English king, Edward III. A globe by Frisius, of 1537, shows the Hudson Bay region, correct as to major characteristics; hence, someone must have made a thorough exploration, and the mapmakers benefited therefrom.

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[1] "An English Scientist in America 130 Years Before Columbus, by H. R. Holand, Transactions, Wisconsin Academy of Sciences, Arts, and Letters, Vol. XLVIII, Madison, Wisconsin, 1959.

SELF-CHECKING METHOD OF COMPUTING CURVE ELEVATIONS[a]

Discussion by Alfred W. Fischer

ALFRED W. FISCHER.[3]—The author has presented a systematic method for analyzing the vertical curve which he explained in his paper. The example given cites an instance when the H. P. falls between the V.P.C. and the V.P.T.

The author made no mention of whether the H.P. falls outside of the V.P.T. However, the writer solved a case for which the H.P. falls outside of the V.P.T., and the author's method gave correct results.

The directions given in Step III are satisfactory when a modern computing machine is used, but Step III must be revised when using an older computing machine that gas only an upper dial (U.D.O.) lower dial (L.D.) and a keyboard (K.B.).

For plus gradients the figures are set into the machine in the following order:

a. The elevation of the V.P.C., 45.63 is placed in the L.D.
b. The station of V.P.C., 86 + 75 is placed in the U.D.
c. The gradient g_1 or + 0.70000 is placed on the keyboard.

With these figures in the machine, the machine is put into operation either by hand or by an electric motor using the plus (+) bar until the station 87+00.00 appears in the U.D. and then the elevation 45.805 will appear in the L.D. (Care must be taken so that the decimals are properly used).

Remove the + 0.70000 from the K. B. and place the g_2 or + 0.6293 in the K. B., operate the machine with the plus (+) bar until station 87+10.283 appears in the U. D. and then the elevation 45.870 will appear in the L. D. This operation will be continued until a $-g_4$ or - 0.08984 is encountered.

With station 88+62.500 in the U. D., elevation 46.333 in the L.D. and 0.08884 in the K.B. reduce, through use of the minus (-) bar, the (U. D) by 44.920 = station (89+07.420 - 88+62.500) or operate the machine until it shows (88+62.500 - 44.920) = station 88+17.580 in the U. D. Then the elevation 46.293 will appear in the L. D. Now clear the K. B. and put station (88+17.580 + 2 x 44.920) = 89.0742 in the U. D. with the plus (+) bar. Repeat this operation for any minus gradients and finally you will have V. P. T. = station 94+75.000 and the elevation 38.830

This method can also be used if a modern computing machine is used and it appears as short as the author's especially for a plus gradient.

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[a] February, 1960, by Charles M. Lamont.
[3] Formerly with the Michael Baker, Jr., Inc., Cons. Engrs., College Park, Md.

[illegible]

INTERSECTION OF STRAIGHT LINE WITH SPIRAL[a]

Closure by T.F. Hickerson

T.F. HICKERSON,[6] F. ASCE.—The inadvertent omission of Fig. 1 from the original manuscript, plus a few typographical errors in its publication[7] tended to obscure its meaning.

The writer is grateful to Mr. Bates for checking the accuracy of the illustrative examples. But in the alleged "intricate procedure for solving a relatively simple problem," Mr. Bates overlooks the fact, already tested, that it is readily adapted to rapid electronic computation. Whereas his "computation of a few triangles by the aid of desk calculator" may not be so simple (or accurate) as the statement implies.

One must remember that the intersections, for example, points on bridge piers, must be located to a high degree of precision, and the method of so doing should be self checking.

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[a] February, 1960, by T. F. Hickerson.

[6] Kenan Prof., of Applied Math., Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

[7] Proceedings, ASCE, ERRATA, Vol. 86, No. SU 2, p. 47.

ERRATA

Journal of the Surveying and Mapping Division

Proceedings of the American Society of Civil Engineers

July 1960

pg. 11-The name;

Robert H. Dodds Acting Chairman

should be added to the list of committee members for this report.

p. 42. In line 2 change Fig. 1 to Fig. 2.

p. 44. In line 9 change 0.4807 ft to 0.4807 min.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c]

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580 (EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596 (HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(SM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(SM5), 2624(EM5), 2625(SM5), 2626(SM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675 (EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692 (ST12), 2693(ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

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JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706 (HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715 (ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2733(PO1)[c], 2724 (HY1)[c], 2725(ST1)[c].

c. Discussion of several papers, grouped by divisions.